Design of Levee Heights for Shallow Aboveground Reservoirs

An analysis performed by EN-H Branch of the Jacksonville District US Army Corps of Engineers, April 2003

Robert C. Tucker, P.E.[1]
Lori L. Hadley[2]

[1] Primary Author, Lead Hydraulic Engineer, Jacksonville District, EN-HH
PO Box 4970, 32232-0019, (904)232-1197,
robert.c.tucker@saj02.usace.army.mil
[2] Co-Author, Coastal Engineer, EN-HC

Table of Contents

Design of Levee Heights
for
Shallow Aboveground Reservoirs

1 Abstract

In South Florida, an ecosystem restoration plan to preserve unique wetlands presented an opportunity to design a continual water supply system to meet both urban and wetlands future demands using shallow aboveground reservoirs. Reservoir levee heights were identified as a critical feature requiring investigation with respect to minimizing litigation risks and protecting public safety, while managing costs within project limits. The design of large reservoirs has been well documented, but the design of shallow reservoirs--less than six feet in depth--has not been so. This lack of reference puts a strain on determining practicality from a depth-storage volume versus cost perspective. Therefore, this paper expresses how hydrologic and hydraulic analyses were performed by Jacksonville District to address Paragraph 7.b and 9.a of ER 1110-8-2 (1991) in determining minimum dam-levee heights for shallow aboveground reservoirs where risk to human life is a possibility should a breach occur. The analyses consisted of selecting an appropriate Probable Maximum Precipitation storm event for routing the Inflow Design Flood; determining wind set up and wave height generation with different methodologies; and calculating the sub sequential wave run up. For a 10% wave run up overtopping rate, resulting levee design freeboards were determined to fall between 11.29 feet and 11.62 feet above 6-foot and 2-foot normal pool stages, respectively. These levee heights were primarily dependent on the selected Probable Maximum Precipitation event and wave run up overtopping probability values.

2 Background

A Comprehensive Review Study of the Central and Southern Florida Project was completed in 1999 with proposed design features that included several shallow aboveground reservoirs (SRsv). During a study, it was determined that SRsv are Federally defined as dams because of their capability of storing greater than 50 acre-feet, thus, making them subject to Corps Regulations (National Dam Safety Act of 1972). The main concern generated by classifying the SRsv as dams is how to determine the minimum levee height where risk to human life is a possibility while managing costs within project limits. Current Regulations prescribe dam-levee height requirements without respect to normal pool depth or direct guidance in selection of the Inflow Design Flood (IDF). Therefore, this paper proposes a methodology based on the analyses results that can be used for all future SRsv to meet Corps Regulations.

3 Analyses Constraints

The C-11 Impoundment was the reservoir chosen with its design parameters defined as the analyses constraints. This restricts the normal pool depth of storage to six feet with control inflow--exception of direct rainfall within the perimeter levee. A total fetch length of 1.57 miles is set with a maximum wind velocity of 120 mph assigned equating to South Florida's Building Code (Category III Hurricane).

4 Analyses Performed

4.1 Hydrologic Analysis

The dynamic surcharge pool elevation is represented by the resulting stage hydrograph found by routing the Inflow Design Flood (IDF) through the reservoir with an emergency overflow spillway. Since the IDF is an explicit function of the Probable Maximum Precipitation (PMP), three methods to estimate the PMP were examined. Also, a matrix of seven spillway lengths and two crest elevations were analyzed for effect on peak stage (maximum surcharge pool) and pool recession rates.

4.1.1 Inflow Design Flood Analyses

Routing the PMP over the watershed above the point where the dam is sited develops the IDF for the case of a classic river dam. Since the C-11 Impoundment has as a constraint that all inflows are controlled, it does not have a watershed area for gravity inflow. Therefore, the watershed used to develop the PMP estimate is simply the internal surface area of the reservoir. The first method of estimating the PMP was for a 6-hour storm event using data provided in the U.S Weather Service's Technical Paper No. 40 (TP-40). For comparison, two discrete methods to estimate the 72-hour PMP were also analyzed. The first method was essentially the extension of the TP-40 from a 6-hour to a 72-hour storm. Data for this approach is provided in the Weather Service's Technical Paper No. 49 (TP-49). The second method is a direct formulation with data provided in the U.S. Weather Service's Hydrometeorological Report No. 51 (HMR-51).

The 6-hour TP-40 PMP was developed using the following procedural steps for point precipitation.

(1) Rainfall depths for the reservoir site were found with use of the TP-40 isopluvial maps of storms with 100-year return period and durations of 0.5, 1, 2,3, and 6 hours. These values were tabulated with intermediate durations of 30 minutes determined through linear interpolation.

(2) TP-40 has an additional map showing the ratio of probable maximum 6-hour precipitation for 10 square miles to the 100-year 6-hour rainfall. This ratio for the site is approximately 4.0, hence, the tabulated values subsequently multiplied by 4.0 to give the PMP values.

(3) Tabulated PMP values are for cumulative rainfall. An incremental distribution table was developed for input into the HEC-1 model for routing of the IDF.

(4) The distribution was then balanced so that the peak increment occurs about midway in 3rd quarter of the storm event. The balance table was then tuned for a smoother curve response.

The 72-hour TP-49 PMP was developed by employing the same step process with inclusion of the 12-hour and 24-hour data from TP-40. The intended use of HMR-51 is to estimate PMP's for large watersheds of 10 to 20,000 square miles using isopluvial maps produced using a standardized Depth-Area-Duration analysis of observed point precipitation amounts collected from major storms. The observed rainfall depths are then increased by a moisture maximization procedure that increases the observed precipitation depths to the maximum theoretically possible. Hydrometeorological Report No. 52 (HMR-52) specifies the stepwise approach to develop the temporal and spatial distribution of PMP estimates derived from HMR-51 maps. The Corps' HMR-52 program automates this procedure and was used to develop input for the HEC-1 program to route the PMP through the reservoir with a resulting stage hydrograph (surcharge pool stages).

In the HEC-1 model, the basin generating the IDF is the interior surface area of the reservoir that receives the PMP rainfall distribution with an elevation-storage relationship. An emergency overflow spillway was provided as an outflow point and variably sized for sensitivity studies later described. For model input, an SCS Curve Number of 99 was used and the lag was simulated by a gravity wave within the reservoir water column--lag = (reservoir longest dimensional length) / (32.2 * depth of normal pool)$^{0.5}$.

4.1.2 Inflow Design Flood Results

The cumulative rainfall depths from the 6-hour TP-40, 72-hour TP-49, and HMR-51 PMP's are 36.4, 68.0, and 55.7 inches with total IDF volumes of 4,569, 8,443, and 6,916 acre-feet, respectively. The additional rainfall from the 72-hour storm results in higher peak stages in the reservoir. However, in recognition of the potential for a looping tropical storm or hurricane that can occur in South Florida, the 72-hour PMP may be considered more appropriate than the 6-hour PMP.

HMR-51 was developed to aid design of projects involving large watersheds (i.e., greater than 1,000 square-miles), thus the "Area" component of the Depth-Area-Duration aspect of the data provided in the report. TP-40 and TP-49 on the other hand provide isopluvial maps based on point rainfall data for all recorded events resulting in a PMP ratio being applied to obtain PMP values. The isopluvial maps provided in the TP-40 and TP-49 are higher in resolution (i.e., more isohyets) for Florida than the maps provided in HMR-51. However, the resolution provided in the TP-40 and TP-49 maps is dampened when the PMP ratio factor is picked from the lower resolution PMP ratio map. The relative lack of resolution in the PMP ratio map reduces confidence for this PMP determination method. The HMR-51 PMP method is therefore recommended even though the small watersheds of the impoundments are more suited to the point rainfall estimates of TP-40 and TP-49.

4.1.3 Uncontrolled Emergency Spillways

Spillway crest elevations were modeled at two heights: (1) pump off--known as the normal pool or 10.00 feet and (2) the height required for containment of the local regulatory storm event (25-year, 72-hour) with respect to pre- and post-development runoff and water quality permitting issues, i.e. 1.2 feet higher than the normal pool--known as the full pool or 11.20 feet. If spillways are designed with a crest elevation at normal pool, all rainfall events that occur when the reservoir is at design depth will generate more runoff than pre-project conditions and the existing flood damage reduction capabilities in the area are reduced. Crest lengths modeled ranged from 50 feet to 5000 feet to limit impact to an expected over-taxed drainage system during times when overflow may occur because of the limited conveyance capacity of discharge receiving canals. For the C-11 Impoundment, Table 1 illustrates an abridged comparison between the two crest elevations for the 50-, 200-, 500- and 2000-foot spillway lengths. It is evident that spillway lengths demonstrate an example of diminishing returns with respect to levee height requirement at the risk of impact to local flood damage reduction capabilities. In summary, large uncontrolled emergency spillways provide limited effectiveness in reducing surcharge or maximum pool elevations, based on all design alternatives and model runs.

4.1.4 Pool Recession and Levee Superiority

Regulations require five feet of levee superiority if the pool stage hydrograph remains within three feet of the maximum pool stage for 36 hours or longer (ER 1110-8-2, 1991, Paragraph 9.c). Because of the watershed constraint for the reservoir and the local regulatory requirement to contain the design flood event, this requirement cannot be met if the regulation is strictly interpreted. For example, if the crest elevation of the spillway is set at 11.20 feet or the full pool as required, and the 72-hour HMR-51 PMP is used, the peak stages range from 13.91 to 12.62 feet for spillway lengths from 50 to 2,000 feet, respectively. Thus the difference in maximum pool above the spillway crest ranges from 2.71 to 1.42 feet, respectively. If only the emergency overflow spillway is used as an outlet, the requirement to bring the pool down three feet cannot be met regardless of the pool recession rate. It may be suggested that gated discharge structures be used to control releases from the impoundment. However, Paragraph 8.d. of ER 1110-8-2 states that: *"Reservoir regulating outlets should not be assumed operable during the occurrence of an IDF, unless they are specifically designed for such purpose."* Even if the gated discharge structures were designed to open automatically under certain pool stages, it is unlikely that this would become part of the water control plan since major releases from the impoundment during such an event would adversely impact local flood damage reduction.

4.2 Hydraulics Analysis

Three naturally occurring actions, wind set up, wave generation, and wave run up are analyzed to determine dam-levee height requirements for containment of levee overtopping. Wind set up (WSU) is defined here as the vertical rise of water on the lee shore of a levee with respect to the still water level in response to wind. Wave generation (WGN) is defined here as the generation of wave height due to wind blowing across the fetch length. Wave run up (WRU) is defined here as the vertical run of water up the lee shore levee in consequence of wave breaking, with respect to the still water level with the addition of WSU. In addition to WSU is wave set up, not to be confused with WRU. Wave set up is defined as that vertical rise of the still water level caused by wave action alone. Wave set up in these analyses is included in the WRU calculations and was not independently studied.

The maximum wind velocity selected for the WSU and WGN analyses was based on South Florida's Building Code requirement of 120 mph, equating to a Hurricane Category III storm. In addition, velocities of 100 and 90 mph representing Category II and Category I storms, respectively, were modeled to test sensitivity of this parameter at high wind values. Realizing that point exposure to very high windspeeds may be of short duration, lower wind speeds may be more practical for levee height design when extending effects to evaluate levee integrity and breach probability.

4.2.1 Wind Set Up (WSU)

WSU occurs when surface shear stress is developed between the air and water interface impelled by wind energy. The amount of vertical rise is limited with respect to opposing forces balanced between surface shear and bottom shear stresses and gravity induced return flow from gained potential energy. This interaction creates a counter-intuitive effect whereby shallower water bodies exhibit a higher degree of wind set up than deeper waters. Three discrete methods of analysis were investigated: the Zeider Zee formula (EM 1110-2-1420, 1997); model developed by Robert Dean and Robert Dalrymple (Dean, 1991); and model presented by Arthur Ippen, Ph.D. (Ippen, 1966). The C-11 Impoundment appears to resemble Ippen's modeled conditions more than the other two; however, the other models have their own merits and were used for comparison.

Fetch length is a sensitive parameter in calculating WSU and WRU. Due to this sensitivity, a cost optimization should be performed for each reservoir to determine the best alignment of interior wind breaks or interior levees to adjust fetch to allow for lower levee heights. For the purposes of this study, fetch was not analyzed independently and is set at 1.57 miles as a constraint for WSU calculations. It is interesting to note that the fetch length is linearly proportional in the Ippen and Zeider Zee models, but lies under the square root in the Dean and Dalrymple model. The method used to calculate wind fetch for this study is referenced in EM 1110-2-1414.

4.2.1.1 Zeider Zee

The Zeider Zee formula was derived from a narrow rectangular water body more typical of a river basin reservoir. The model is empirical and dimensionally incorrect with no limitations or boundaries of solutions. The model is often used because it is simple, straightforward and provides conservative values when solved. Results for the C-11 Impoundment are illustrated in Figure 1. Note that Zeider Zee's solution rises exponentially as depths change from deep to shallower water. This curve is attributed to the empirical nature of the model and is not considered applicable to shallow impoundments with depths of six feet or less.

4.2.1.2 Dean and Dalrymple

The Dean and Dalrymple model (D&D) was derived from data collected along the continental shelf that may be characterized by relatively shallow water that provides an infinite volume of water for WSU originating from deeper waters. The model's equation differs from Zeider Zee primarily by the incorporation of wind shear stress as a parameter. Results for the C-11 Impoundment are illustrated in Figure 1. The graph illustrates that the D&D model calculates the highest WSU for water depths greater than three feet. See cited reference for a description of the model equations and their limitations (Dean and Darylmple, 1999).

4.2.1.3 Ippen

Ippen's model was derived from data collected on shallow, enclosed, regular-shaped reservoirs. It differs from the others in that it takes into consideration when volume of water is limited in determining the WSU. The model's integral equation is based on surface and bottom shear stresses and conservation of volume mass. Results for the C-11 Impoundment are illustrated in Figure 1. Use of Ippen's model invokes a limitation for determining a numerical solution of the model's integral equation. Solutions were not found for the one-foot depth or depths deeper than ten feet or more, depending on wind velocity as a varying parameter. In the extreme shallow depth range, the model appears to become unstable. For depths deeper than 11 feet, the modeled curve appears to converge with that of Zeider Zee. See cited reference for a description of model equations and their limitations (Ippen, 1966).

4.2.1.4 Recommended Method

Noting the results from the three reviewed models, it appears that the best methodology is one that amalgamates the Zeider Zee and Ippen models to best represent the initial set of conditions of applicability. The overall effect would desensitize Zeider Zee's exponential break for shallow water depths and Ippen's instability for very shallow water depths. A modified method (mod-T) is proposed here with a notice that it is presently an undocumented method that has not been verified with measured or lab data. Results for the C-11 Impoundment are illustrated in Figure 1. The mod-T method is described in the following two-step approach.

(1) Calculate Ippens's deepest depth with a solution and compare the WSU with that determined using Zeider Zee at the same water depth. Add the difference to the full range of Ippen's water depth WSU solutions.

(2) Determine the steepest slope demonstrated by Ippen's WSU solution, from deep to shallow. The slope is then aligned with the mod-T WSU solution from point of departure from rising curve and extended to desired WSU solution depth, again, from deep to shallow.

4.2.2 Wave Generation (WGN)

Wind waves, also known as oscillatory waves, are most commonly defined by their height, length, and period. These characteristics, as measured at a given location, are determined by five dominant factors: wind velocity or speed, effective fetch distance, duration over which the wind blows (considered to be unlimited for study purposes), decay distance the wave travels after leaving the generating area (not relevant to this study), and the water depth. The process of wind wave growth (assuming initial still water) begins with the motion of the air above the water disturbing the surface of the water leading to the formation of small perturbations in the water surface. When the perturbations become large enough to affect the pattern of air flow a transfer of momentum and energy between the air layer and the water surface occurs, rapidly increasing wave heights. The faster the air layer moves (i.e. the greater the wind speed), the more momentum is transferred into the developing wave field. In the early stages of wave growth the waves are essentially deep water waves and do not feel the effects of the bottom. In shallow water with high wind velocities, wave growth quickly achieves the physical characteristics and water-bottom interaction that limits subsequent growth. If the wind duration exceeds the time required for the waves being generated to travel the entire fetch length, the waves will grow along the fetch and their characteristics at the end of the fetch will depend only on the fetch length and wind velocity. This is a "fetch limited" condition.

Two different numerical methodologies determining wave growth and water bottom interaction were utilized in the analyses: (1) USACE Automated Coastal Engineering System (ACES) and (2) Simulating Waves Nearshore (SWAN). The results from the two models cannot be directly compared because ACES used a constant depth over the entire effective fetch, whereas SWAN used variable depth capability to examine sensitivity of wave growth over an idealized variable bathymetric case. Neither model included wave diffraction, wave set-up, or wave induced currents. All wind observations were assumed taken 10 m off natural grade.

There appears to be some issues to the applicability of "effective" fetch for relatively long and narrow water bodies, with two reasons given for its source: (1) lateral spread of wave energy dissipation on nearby shorelines and (2) an inherent cosine directional spread for wind input. The Shore Protection Manual (SPM, 1984) notes that users of SMB wind growth curves for narrow fetch conditions need to calculate an effective fetch to determine expected wave heights. However, the wind growth curves provided in the SPM do not require this calculation and suggests use of the "straight-line" fetch. Noting that ACES documentation demonstrates the use of calculating an effective fetch with the radial method, an effective fetch was calculated for the wave growth analyses. The effective fetch distance for a relatively long and narrow water body can be as small as 50% of the overall fetch distance. Since the effective fetch distance for winds blowing diagonally across rectangular-shaped impoundments should be less than the overall fetch distance, but more than that for a long and narrow water body, a geometric method of calculating this distance was implemented. This method accounted for an approximate 30% reduction from the total fetch length..

4.2.2.2 Automated Coastal Engineering System (ACES)

In ACES, estimates for wave growth in shallow water are based upon the fetch-limited deepwater formulas, but modified to include bottom effects with respect to friction or shear stresses (shallow water waves). The formulas for wave growth in deep water encompass the effects of fetch and duration: however, the modifications based on Bretschneider and Reid (1954) do not utilize duration-limited effects. The unmodified open-water formulas are taken from Vincent and are based upon the spectrally based results given by Hasselmann et al. (1973, 1976). Tabulations of formula usage are found in the SPM (1984). In all cases, the wave growth estimates are bounded by the expressions for a fully developed equilibrium spectrum. ACES documentation stipulates that the relationships have not been verified and may or may not be appropriate for the conditions and assumptions of the original Bretschneider-Reid work.

Model Input

Vincent maintains that wind speed should be adjusted to consider the nonlinear effect on the wind stress creating the waves. The ACES model allows six options in selecting the observation type or manner in which wind was measured (e.g. at sea, onshore wind, etc). The C-11 Impoundment is found inland and the “over land” option was selected. This selection enables the wave growth formula to use the full planetary boundary layer in the modeled solution domain.

Bathymetric input for wave transformation studies ideally represents actual conditions for the region of interest. However, the ACES model does not allow for bathymetric changes for shallow water; therefore, the bottoms are assumed level and smooth.

ACES Results

See Figure 2 for the 7-foot pool depth-120 mph wind case as reference for the ensuing discussion. The ACES model calculates wave height as a single value solution. To examine wave growth along the fetch, multiple single-runs were made, thus, only trends are discussed versus accurate wave height expectations. The transition from early stage of wave growth—necessarily deepwater waves—into shallow waves is smooth with no indicative break point. This is characteristic of the simplified mathematical expressions incorporated by the model. Independent of pool depth, wave heights build rapidly over a short distance, increasing from 0 to approximately 1.2 feet within 700 feet of fetch. Beyond 700 feet, it is expected that deepwater growth transcends to shallow water wave growth (i.e. influence of a bottom is felt by the physical wave being generated with time and fetch). Wave heights continue to increase as the fetch increases, but, as Figure 2 indicates, will not grow unchecked. If the fetch and duration are sufficiently large, the curve becomes essentially horizontal at the downwind edge and a "fully developed sea" has been generated for the particular wind velocity. Increasing wind speed and/or water depth will increase the distance along the fetch required before "a fully developed sea" can form. Table 2 provides the maximum significant wave heights for pool depths of 3, 5, 7, 9, and 11 feet.

4.2.2.3 Simulating Waves Nearshore (SWAN)

SWAN is a third-generation numerical wave model for the realistic estimation of wave parameters in coastal areas, lakes, and estuaries. For given bathymetric, wind, and current conditions, SWAN generates wave parameters based upon the action balance equation (Ris et. al., 1997). This approach includes wave energy generation though wind and current input; wave energy dissipation through bottom friction, whitecapping, and depth induced wave breaking; and wave energy redistribution by non-linear wave-wave interactions. The transfer of wind energy and momentum to waves is represented with a combination of linear and exponential wave growth. The linear term acts as a filter to eliminate growth at frequencies lower than the Pierson-Moskowitz frequency (Tolman, 1992) and the exponential term accounts for the interaction between the wind and waves at the air-water interface dependent on friction velocity, phase speed, and the densities of both air and water. The SWAN model derives a solution through the propagation of energy in geographic space. Dissipation during the growth process is represented by the sum of three different contributions, whitecapping, bottom friction, and depth induced wave breaking. Whitecapping losses are represented by a pulse based model (Hasselmann, 1974) with a dependence on wave steepness. Bottom friction dissipation is determined from a semi-empirical expression derived from the JONSWAP (Joint North Sea Wave Project) results (Hasselmann et al., 1973). Wave breaking losses are simulated as dissipation of a bore applied to the breaking waves in a random field (Battjes and Janssen, 1978) with a variable wave breaking parameter developed by Nelson (1987).

Model Input

Bathymetric input for wave transformation studies ideally represents actual conditions for the region of interest. However, the generalized nature of this analysis does not require actual bathymetric conditions. Instead, a series of furrows aligned perpendicular to the fetch are generated to simulate a non-uniform bottom topography. The furrows, measuring 100ft in width are separated by 50ft intervals over the entire fetch. Each furrow is assigned a depth two feet greater than the designated pool depth for the impoundment. Nine pool depths were modeled, ranging from 3ft to 11ft, with 5ft to 13ft furrows respectively.

In this study, SWAN is implemented on a regular 170 cell by 120 cell Cartesian grid with a cell resolution of 50ft. This results in a total model fetch length of 1.6 miles and a cross-fetch distance of 1.0 mile. As with bathymetry, dimensions of the grid are not meant to simulate the actual dimensions of the C11 impoundment. Instead, dimensions are extended such that boundary effects intrinsic to the model do not influence model results at the point of interest (i.e. at a fetch distance of 1.1miles).

SWAN Results

See Figure 3 for the 7-foot pool depth-120 mph wind case as reference for the ensuing discussion. Noted at the beginning of this section, waves formed in the early stages of wave growth are essentially deep water waves and do not feel the effects of the bottom. This is evident in the smoothness of the upwind portions of the curves corresponding to shortest fetch lengths. Independent of pool depth, wave heights build rapidly over a short distance, increasing from 0 to approximately 0.9 feet within 100 feet of fetch. Beyond 100 feet, non-linear and dissipative effects become increasingly dominant in determining the rate and magnitude of wave growth. Peaks and troughs begin to appear as the effects of the bottom are felt. The influence of bottom friction can also be seen in the decreased sensitivity of wave height to bottom topography as water depth increases. As a result, rapid changes in wave height corresponding to the "furrows" in the bathymetry become less pronounced. Wave heights continue to increase as the fetch increases, but, as the figure indicates, will not grow unchecked. If the fetch and duration are sufficiently large, the curve becomes essentially horizontal at the downwind edge and a "fully developed sea" has been generated for the particular wind velocity. Increasing wind speed and/or water depth will increase the distance along the fetch required before "a fully developed sea" can form. Sea development can be clearly seen in both figures. It should be noted that the irregular bathymetry and incorporation of nonlinear wave-wave interactions allows for some amount of continued wave growth rather than the achievement of a static equilibrium condition.

Wind wave growth is also checked by wave breaking. Instances of wave breaking are marked by sudden drops in wave height, followed by a regeneration of the wave field. Traditionally, wave breaking is determined by the expression $H_b = \gamma d$, where H_b is the breaking wave height, γ is the breaker parameter (typically a constant), and d is water depth. However, SWAN incorporates Nelson's formulation (Nelson, 1987), which varies the breaker parameter based on the slope of underlying bathymetric features. This results in variable breaking wave heights. Despite the variability in wave height, the regularity of the "furrowed" bathymetry results in nearly consistent wave breaking at approximately 4,200 feet along the fetch. Model results also reflect that wave breaking is influenced by water depth, becoming less pronounced as pool depth increases. Table 2 provides the maximum significant wave heights for pool depths of 3, 5, 7, 9, and 11 feet.

4.2.3 Wave Run Up (WRU)

Wave run up can be described as the resulting forward translation of water mass that is converted from wave energy stored as rotational or oscillatory motion into potential energy as water "runs" up a barrier face. WRU depends primarily on structure shape and roughness, water depth at structure toe, and incident wave characteristics (SPM 1984). Generally, for a given incident wave height and period, the steeper the levee slope the higher the WRU because of a combination in potential energy gained and energy dissipation through levee face friction. For this reason, slopes investigated for WRU in this analyses set were 1:3, 1:3.5 and 1:4 (rise:run). All WRU calculations have the following assumptions: (1) waves are irregular and characterized by the significant wave height, (2) waves approach is perpendicular to structure, and (3) structure shape is an impermeable planar slope. Only results stemming from the 120 mph wind velocity case are provided in this paper.

4.2.3.1 Method Selection

The SPM (1984) provides a graphic means to calculate expected wave run up on a structure slope given deepwater significant wave height, wave period, and water depth at structure toe. It is important to note that the graphs (Figures 7-8 through 7-12) are drawn from model experiments and requires correction for prototype estimations (Figure 7-13).

4.2.3.2 Slope Variable Roughness

Graphic charts found in the SPM (1984) are for smooth surface slopes; however, quantification of WRU is sensitive to surface roughness. There are various methods of reducing WRU with roughness integration with structure slopes (typically to protect the structure). For example, the SPM provides a reduction factor of 0.85 to 0.90 for grassy slopes and 0.80 for one layer of quarrystone randomly placed. However, for public safety concerns and with the condition of grass-lined slopes questionable at times, a smooth surface was implemented for the main analysis to produce a conservative maximum WRU quantification.

4.2.3.3 Irregular Waves

Wind generated waves are irregular, covering a variable range of heights and periods. The WRU analysis indicates only to what vertical height WRU may be expected to occur based on the significant wave height, Hs, or the highest 33% of the waves, given the spectrum of waves generated. Ultimately, the purpose of analyzing WRU is to identify the probability of overtopping and the rate of overflow that may occur. The ability of the levee to resist erosion on the lee face is a function of Geotechnical analyses of the proposed levee material and construction technique; therefore, it is not included here. However, analysis of the WRU based on percentage probabilities of 1%, 2%, 10%, 20% and 33% was performed with SPM Equation 7-9 (1984).

4.2.3.4 Wave Run Up Results

Results of the WRU analysis are illustrated in Tables 2 and 3. Review of Table 2 illustrates that the WRU difference between the 1:4 slope and the 1.3 slope is approximately 75%. If levee material is at a premium, such as in South Florida where limestone is blasted when excavating borrow and subsequently ground to specifications, a lower slope or levee bench may not be acceptable and should be addressed by a cost optimization analysis. Review of Table 3 provides a quick insight that the expected height reached by the top 1% of WRU is twice that of the top 33%. This illustrates the importance of determining the acceptable overtopping rate that a levee may tolerate without risk of breach.

5 Conclusion

This report expressed how the Jacksonville District examined hydrologic and hydraulic phenomenon affecting the design of levee heights for shallow aboveground reservoirs under harsh weather conditions where risk to human life is a possibility should a breach occur. The resulting numerical calculations should not be taken as absolute because of real site variations differing from model conditions as demonstrated between the different methodologies used in the analyses. Most assumptions required for model input were made conservatively with respect to an interest for public safety. It is anticipated that the following unaccounted for site conditions will affect actual realized WSU and WRU occurrence: actual rainfall volume versus time distribution, vegetation (bottom friction), wave reflection off levee faces, and actual bathymetry. Also, for shallower impoundments vegetation may trap a significant volume of water that would otherwise be available for WSU.

It is commonly mentioned that it is rare for a high wind producing storm event to occur with very heavy precipitation. However, in South Florida where the potential for a looping Hurricane, or a stationary one is a real possibility, the IDF must figure directly into the overall levee height requirement. With these points stated, Table 4 presents numerical results for the 120 mph wind speed case and recommends the stated levee height where a breach would put public safety at risk. The IDF peak stage for the 72-hour HMR-51 PMP with a 200-foot emergency overflow spillway was set at 3.5 feet versus the PMP 4.6 feet with no spillway (see Table 1). This is considered practical even if the emergency overflow spillway should provide little relief because of an assumed indeterminate amount of volume of water escaping via spray, overtopping and seepage.

Results shown in Table 4 illustrate that under extreme meteorological events, there is a significant difference between ER 1110-8-2 (1991) requirement and what South Florida may design in consideration of risk to human life. For the 10% overtopping rate, difference ranges from—assuming the Regulation requirement of IDF peak stage plus five feet freeboard—2.8 feet higher (6-foot pool) to 3.1 feet (2-foot pool) for adequate design. For 20% overtopping, the difference drops approximately a foot. The results also illustrate the significance of WRU when compared to the other contributing parameters; therefore, it is strongly suggested that actual bathymetry and bottom friction be included in a wave generation model, such as SWAN, to evaluate WGN accurately with sub sequential WRU. Levee surface roughness may also be evaluated to determine if additional features (e.g. quarrystone lining) may be cost effective to lower WRU.

A suggested procedure to determine levee heights is presented here with notes of importance to be considered when devising a calculation strategy. First step is to select an appropriate PMP event and route the IDF to determine the peak stage (maximum surcharge pool). Locating the emergency overflow spillway is important. If the spillway is not located on or near the levee lee to the critical fetch (i.e. public safety), the spillway may offer little in way of relief of higher pool stages with simultaneous occurrence of high wind speeds. Second step is to use an appropriate WSU model to determine the pool depth for WGN with sub sequential WRU quantification. If levee material is at a premium with respect to cost, it is further recommended that a geotechnical analysis be performed to determine the cost effective levee slope and maximum overtopping rate that proposed levee material and construction technique will provide, minimizing levee cross-section area and risk of a levee breach.

References

Battjes, J.A. and J.P.F.M. Janssen (1978). Energy Loss and Set-up Due to Breaking of Random Waves, *Proceedings 16th International Conference on Coastal Engineering, ASCE*, 569-587

Bretschneider, C.L. and Reid, R.O. (1954). Change in Wave Height Due to Bottom Friction, Percolation and Refraction, *34th Annual Meeting of American Geophysical Union*, 1953

Brigham Young University Engineering Computer Graphics Laboratory (1997). *Surface-Water Modeling System Reference Manual*, Brigham Young University, Provo UT

Dean, R.G. and Dalrymple, R.A. (1991). *Water Wave Mechanics for Engineers and Scientists*, World Scientific, New Jersey

EM 1110-2-1420 (1997). *Hydrologic Engineering Requirements for Reservoirs*

EM 1110-2-1414 (1989). *Water Levels and Wave Heights for Coastal Engineering Design*

ER 1110-8-2(FR) (1991). *Inflow Design Floods for Dams and Reservoirs*

Hasselmann, K. (1974). On the Spectral Dissipation of Ocean Waves Due to Whitecaping, *Boundary Layer Meteorology*, 6, 1-2, 107-127

Hasselmann, K, T.P. Barnett, E. Bouws, H. Carlson, D.E. Cartwright, K. Enke, J.A. Ewing, H. Gienapp, D.E. Hasselmann, P. Druseman, A. Meerburg, P. Muller, D.J. Olbers, K. Richter, W. Sell, and H. Walden (1973). Measurements of Wind Wave Growth and Swell Decay During the Joint North Sea Wave Project (JONSWAP), *Dtsh. Hydrog*. Z. Suppl., 12, A8

Ippen, A.T. (1991). *Estuary and Coastline Hydrodynamics*, Bretschneider, C. L: McGraw-Hill Book Company, Inc., New York

National Dam Safety Act of 1972, Public Law 92-367 and 104-303, Section 215

Nelson, R.C. (1987). Design Wave Heights on Very Mild Slopes: An Experimental Study, *Civil Engineering Trans*., Inst. Eng., Aust.29, 157-161

Ris, R.C., N. Booij, and L.H. Holthuijsen (1997). *SWAN Cycle 2, User Manual*.

Tolman, H.J. (1992). Effects of Numerics on the Physics in a Third Generation Wind-Wave Model, *Journal of Physical Oceanography*, 22, 10, 1095-1111

U.S. Army Engineer, Waterway Experimental Station. (1984). *Shore Protection Manual*, USPO Washington D.C.

U.S. Weather Service, *Hydrometeorological Report No. 51*, Probable Maximum Precipitation Estimates, United States East of the 105^{th} Meridian

U.S. Weather Service, *Hydrometeorological Report No. 52*, Application of Probable Maximum Precipitation Estimates, United States East of the 105^{th} Meridian

U.S Weather Service, *Technical Paper No. 40*, Rainfall Frequency Atlas of the United States for durations from 30 Minutes to 24 hours and Return Periods from 1 to 100 Years

U.S. Weather Service, *Technical Paper No. 49*, Two to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the United States

Figures and Tables

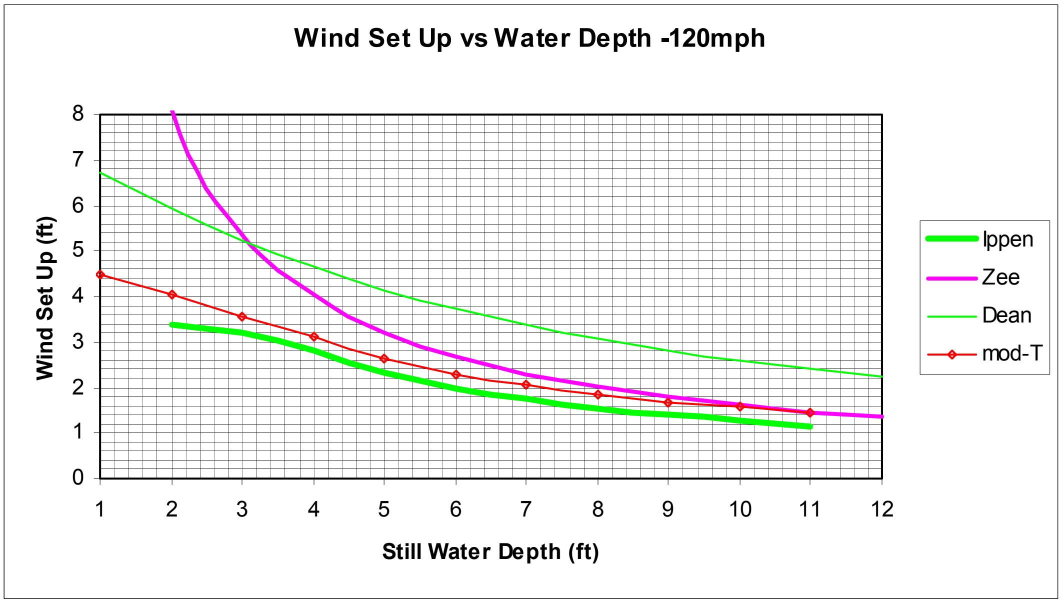

Figure 1 – Wind Set Up

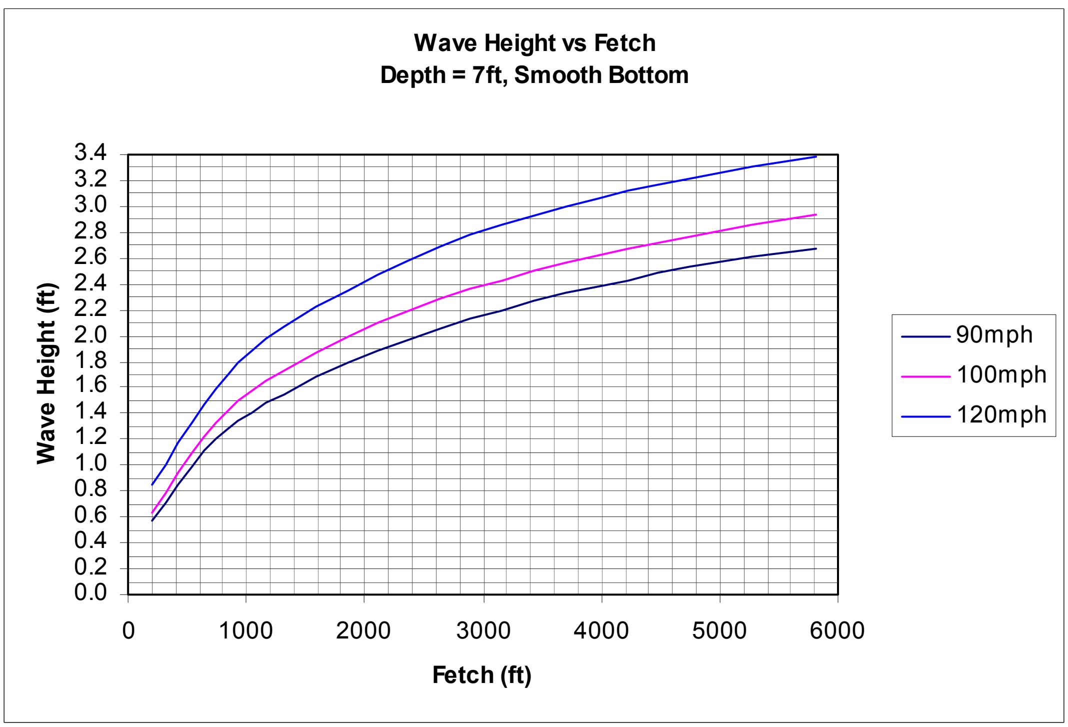

Figure 2 – ACES Wave Generation

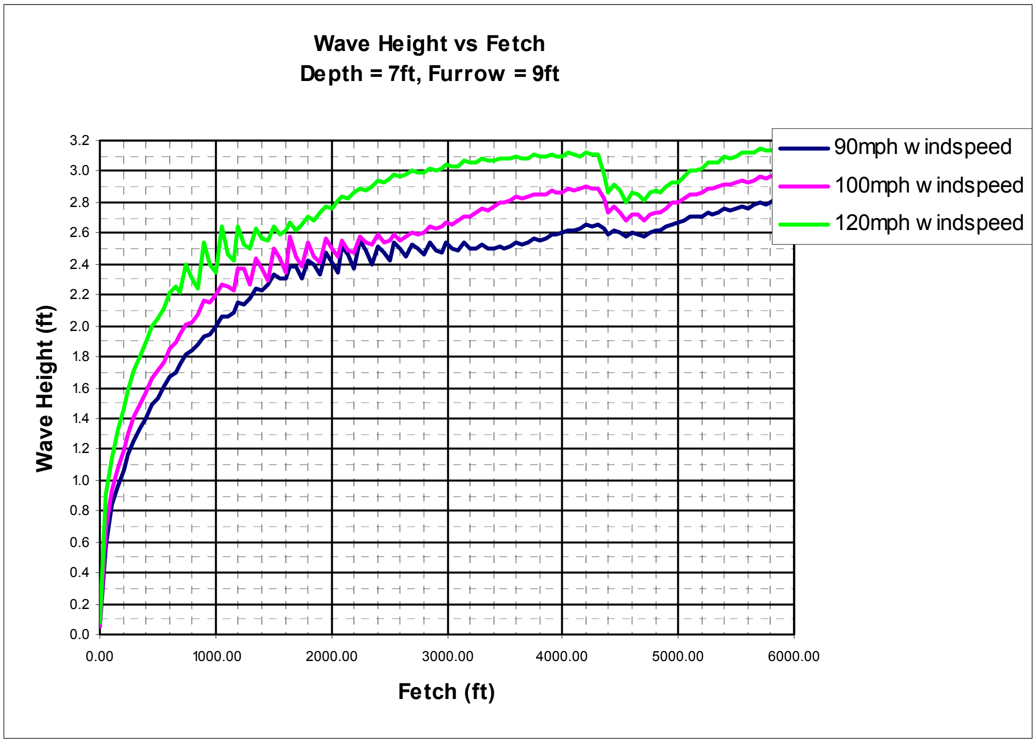

Figure 3 – SWAN Wave Generation

Table 1 – IDF Results with Spillway

Spillway Length	6-hr TP-40				72-hr HMR-51			
	Crest Elevation (feet)		Peak Discharge (cfs)		Crest Elevation (feet)		Peak Discharge (cfs)	
	10.00	**11.20**	**10.00**	**11.20**	**10.00**	**11.20**	**10.00**	**11.20**
50	12.95	13.02	761	368	13.64	13.91	1046	671
200	12.70	12.91	2664	1343	13.03	13.54	3159	2158
500	12.38	12.74	5537	2873	12.43	13.21	5710	4295
2000	11.73	12.39	13,690	7771	11.53	12.62	11,402	10,251

Table 2 – Wave Generation and Run Up Slope Results

Still Water Depth	Wind Set Up, WSU	Max Wave Height for the Hsig Wave (feet)		Wave Run Up, ACES, Hsig Smooth Levee Face, (feet)		
(feet)	(feet)	ACES	SWAN	3.0 Cotan	3.5 Cotan	4.0 Cotan
3.0	3.57	2.05	1.53	3.50	3.03	2.57
5.0	2.65	2.89	2.36	4.42	3.77	3.30
7.0	2.05	3.37	3.16	5.05	4.32	3.81
9.0	1.69	3.68	3.83	5.42	4.71	4.09
11.0	1.46	3.89	4.13	5.72	4.87	4.28

Table 3 – Wind Wave Run Up Probability

Top Percent Exceedance	Wave Run Up, 3.0 Cotan Smooth Levee Face, (feet)			
%	5-foot	7-foot	9-foot	11-foot
1	6.71	7.66	8.22	8.68
2	6.18	7.06	7.58	8.00
10	4.74	5.42	5.82	6.14
20	3.97	4.53	4.86	5.13
30	3.29	3.76	4.04	4.26

Table 4 – Required Levee Height Results

Normal Pool Depth	HMR-51 PMP	Wind Set Up (mod)	Wave Run Up, Slope = 1:3		Required Levee Ht
			Exceed	Smooth	
(feet)	(feet)	(feet)	%	(feet)	(feet)
2.0	3.5	2.50	2%	7.32	15.32
			10%	5.62	13.62
			20%	4.69	12.69
3.0	3.5	2.18	2%	7.50	16.18
			10%	5.76	14.44
			20%	4.81	13.49
4.0	3.5	1.95	2%	7.67	17.12
			10%	5.89	15.34
			20%	4.92	14.37
5.0	3.5	1.77	2%	7.85	18.12
			10%	6.02	16.29
			20%	5.03	15.30
6.0	3.5	1.63	2%	8.03	19.16
			10%	6.16	17.29
			20%	5.15	16.28